Feb. 28

IJASCSE, Vol 2, Issue 1, 2013

Theoretical study of axially compressed Cold Formed Steel Sections

Sreedhar Kalavagunta , Sivakumar Naganathan, and Kamal Nasharuddin Bin Mustapha
Universiti Tenaga Nasional,
Jalan IKRAM-UNITEN, 43000,
Department of Civil Engineering,
Kajang, Selangor, Malaysia.

Abstract— Conceptual and finite element analysis oriented design of cold formed steel columns are presented in this paper. Total four 1 meter channel lipped section with thickness of 1, 1.2, 1.5 and 1.9 are tested. All columns were tested under a pure axial load, Conceptual design results are compared with finite element analysis for axially loaded compression members. The results provide useful information regarding allowable load estimation of cold formed steel column section. Based on the results, design recommendations were proposed. The proposed design approach is recommended for the design of complex shape of cold formed steel section, where design rules are not available in standards.

Keywords-Cold formed steel, Axial Stress, Design Strength, Effective Width Method, Direct Strength method.

I. INTRODUCTION

Cold Formed Steel (CFS) members are widely used in building constructions, bridge constructions, storage racks, highway products, drainage facilities, grain bins, transmission towers, car bodies, railway coaches and various types of equipment. These sections are cold formed from carbon or low alloy steel sheet, strip, plate, or flat bar in cold-rolling machines or by press brake or by bending brake operations. The thicknesses of cold formed steel members do usually range from 0.378 mm to 6.35 mm. CFS sections gained special attention in research due to its major advantages like flexibility in drawing to any shape, high strength and stiffness, ease of prefabrication and mass production, fast and easy erection, installation etc.

In the recent decades, the use of Cold formed steel has increased widely in the construction industry because of its unique characteristics and advantages. The behaviors of CFS structures are influenced by effects such as Local Buckling, Distortional Buckling and Global Buckling [1, 2] which arises due to the Slenderness of members. This makes the design and analysis often more complex. The recent research and investigation on cold formed steel shows advancements in design specification and manufacturing processes for most of the standard sections [3,4,5,6,7].

The Effective Width Method and Direct Strength Method (DSM) are the only two basic designs that are accessible now by the design codes including North American Specification for Cold-Formed Steel Structural members (AISI-2007) [8]. Effective Width method can take into account the interaction between Local and Lateral buckling. However, as structural shapes became more complex with additional lips and intermediate stiffeners, the accurate computation of the effective widths of individual elements of the complex shapes becomes more difficult and inaccurate. In order to overwhelm this problem, the Direct Strength method (DSM) was developed. However, finite element analysis like ANSYS, ABAQUS etc. are the best options for the design of CFS with any cross sectional shapes, but most of the structural consulting firms are using basic FEA software's like STAAD.Pro, SAP2000, RAM Structural System, STRAP etc., This paper presents a comparative study of cold formed steel column

section design using STAAD.Pro, FEA results with conceptual design as per AISI.

II. MATERIALS AND METHODS

Cold formed steel sections are usually thinner and have a mode of failure and deformation, which are not commonly encountered in normal structural steel design. A thin walled member under compression loads most likely undergoes various buckling modes like local buckling, Distortional buckling, Flexural buckling, Torsional buckling, Flexural-Torsional buckling. For a typical C-Shape column under pure axial compression, the Local Buckling mode is the dominant mode. However, a small change from this prototype, e.g., the addition of lip stiffeners and web stiffeners, can markedly increase the local buckling stress and make the distortional buckling mode dominant, as indicated by Schafer[9].

$$P_n = A_e F_n \tag{1}$$

where

P_n = Design compressive force

A_e = Effective cross sectional area

F_n = Design Stress of Cold Formed Steel.

Thin walled CFS sections can be used efficiently as structural members of light-weight structures when hot-rolled sections or others are not efficient. Until recently, the conventional Effective Width method (EWM) has been the only way to estimate the member strength from the past 60 years. This method accounts the interaction between local and the post-buckling strength. Nevertheless, as structural shapes become more complex with additional lips and intermediate stiffeners, the accurate computation of the effective widths of individual elements of the complex shapes become more difficult and inaccurate. In order to overcome this problem, the Direct Strength method (DSM) was developed by Shafer and Pekoz [10] in 1998 and has been studied further by Hancock at al., [11].

Since the thickness of individual plate elements of CFS structural members are normally small compared to their width, buckling and post buckling strength are two major concerns for strength prediction of CFS structural members. Unlike hot rolled members, CFS members normally buckle prior to section yielding. Further, CFS compression elements do not collapse when the buckling stress is reached. Additional load can be carried out by the element after buckling called as post buckling strength, by means of stress redistribution [12].

In this Effective Width approach [8], instead of considering the non-uniform distribution of stress across the width of the element, it is assumed that the total load is carried by a fictitious width b, which is subjected to a uniformly distributed stress f_{max}. Where, f_{max} equals the edge stress. Professor George Winter at Cornell university proposed a formula to determine post buckling strength of a stiffened element that appeared in AISI known as "Winter's equation".

$b = w$, for $\lambda \leq 0.673$

$b = \rho w$, for $\lambda > 0.673$

where ρ = reduction factor

$$\rho = \left(1 - \frac{0.22}{\lambda}\right) / \lambda \leq 1 \tag{2}$$

Where λ = plate slenderness factor

$$\lambda = \sqrt{\frac{f}{f_\alpha}} = \frac{1.052}{\sqrt{k}}\left(\frac{w}{t}\right)\sqrt{\left(\frac{f}{E}\right)} \tag{3}$$

Where k = plate buckling coefficient;

t = thickness of compression element;

E = modulus of elasticity

f_{max} = maximum compressive edge stress in the element

($f_{max} = f_y$, the yield stress for maximum capacity)

For more complicated configuration, the effective width must be determined for each compression portion, and then the strength of the section can be obtained by assuming load is resisted only by the effective areas.

The Direct Strength Method [8] is initially proposed in 1988 and has been adopted by the North American Cold-Formed Steel Specifications in 2004 as an alternative to the traditional Effective Width Method to estimate the compression and the flexural member strength, which can consider interaction of local or distortional and overall buckling modes. This method does not require effective width calculations or iteration, but as an alternative uses gross properties and the elastic buckling behavior of cross section to calculate section or member strength. With help of software applications, this design procedure can be implemented for any type of sections.

The Direct Strength Method uses the entire cross-section in the elastic buckling determination and offers specific provisions for Local, Distortional and Global buckling strength respectively. The design strength P_n (4) to (6) is determined as minimum of (P_{ne}, P_{nl} and P_{nd}) based on DSM method.

Flexural, Torsional, or Torsional-Flexural Buckling (P_{ne})

$$P_{ne} \begin{cases} \left(0.658^{\lambda_c^2}\right) P_y & \lambda_c \leq 1.5 \\ \left(\frac{0.877}{\lambda_c^2}\right) & \lambda_c > 1.5 \end{cases} \qquad (4)$$

where

$$\lambda_c = \sqrt{P_y / P_{cre}}$$

P_{cre} = Minimum of the critical elastic column buckling load in Flexural, Torsional, Or Torsional-Flexural Buckling.

Local Buckling (P_{nl})

$$P_{nl} = \begin{cases} P_{ne} & \lambda_l \leq 0.766 \\ \left[1 - 0.15\left(\frac{P_{crl}}{P_{ne}}\right)^{0.4}\right]\left(\frac{P_{crl}}{P_{ne}}\right)^{0.4} P_{ne} & \lambda_l > 0.766 \end{cases} \qquad \text{--(5)}$$

Where

$$\lambda_l = \sqrt{P_{ne} / P_{crl}}$$

P_{crl} = Critical elastic local column buckling load.

Distortional Buckling (P_{nd})

$$P_{nd} = \begin{cases} P_y & \lambda_d \leq 0.561 \\ \left[1 - 0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right]\left(\frac{P_{crd}}{P_y}\right)^{0.6} P_y & \lambda_d > 0.561 \end{cases}$$

.... (6)

Where

$$\lambda_{cd} = \sqrt{P_y / P_{crd}}$$

P_{crd} = Critical elastic distortional column buckling load.

Four cold formed lipped channels of C10010, C10012, C10015 and C10019 were considered in this investigation. The American specification (AISI 2007) design concepts used, four finite element models created with sectional properties that are tabulated in table 1.

The column was modeled with 3D shell elements (Figure 1) with sharp corners neglecting the corner radius with applied force of 24.3 kN. Boundary condition is fixed at the bottom and released at top for vertical transaction to capture local bucking. Both conceptual design and FEA analysis were carried out by using STAAD.Pro product. Provisions of the AISI Specification for the Design of Cold-Formed Steel Structural members have been implemented in STAAD Pro.

This Program allow design of single members in tension, compression, bending and shear. Figure 2 and 3 illustrates stress contours and buckling modes.

TABLE I. Test cases of Lysaght Cold Formed Sections

Specimen	**Area mm²**	**Depth mm**	**Width mm**	**Thickness mm**	**Lip mm**	**R mm**
C10010	216	102	51	1	12.5	5
C10012	258	102	51	1.2	12.5	5
C10015	323	102	51	1.5	13.5	5
C10019	409	102	51	1.9	14.5	5

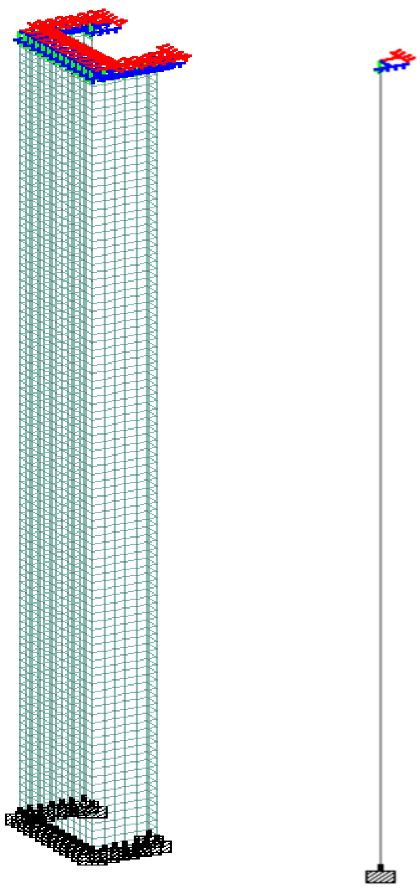

Figure 1: FEA shell element model and Conceptual Beam element model for testing.

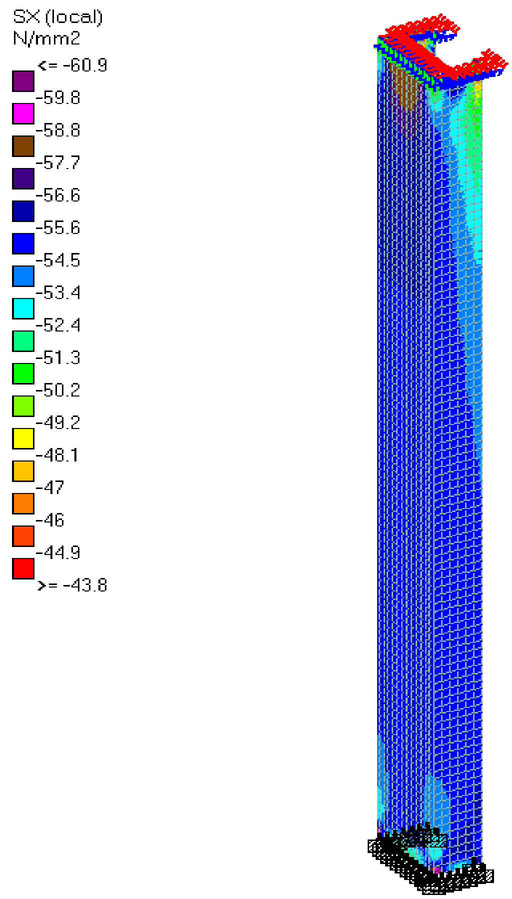

Figure 2: FEA mode with Stress contour and Mode

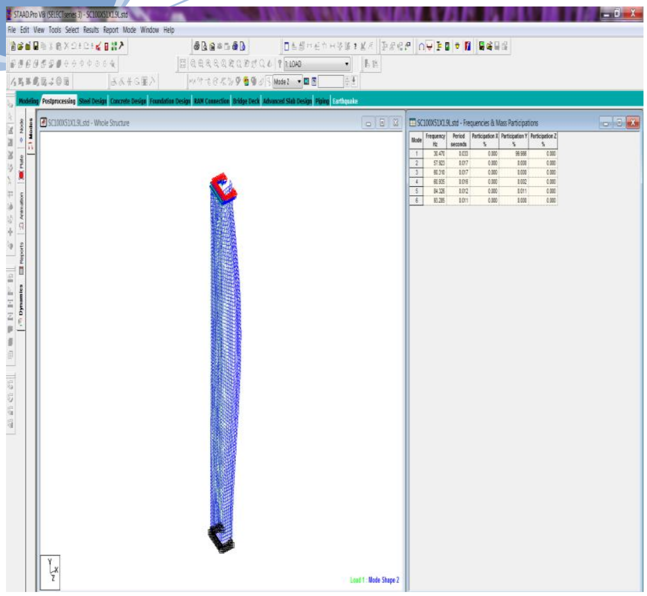

Figure 3: FEA mode with Stress contour and Mode shape.

III RESULTS AND DISCUSSION

The critical stresses from FEA model and conceptual design results as per AISI code provisions are recorded in Table 2 using STAAD.Pro (Structural analysis and design software). The membrane stress Sx, Sy, inplane shear stress Sxy, Shear stress Sqx, Sqy (Force/ unit length/ thickness) and moments Mx, My, Mxy (Force x Length/length) are obtained from FEA analysis. Actually, stresses are calculated based on equations 7-11 and the results are tabulated in Table 3.

Shear Stress in x-direction

$$Sxc = (Sx+Sxb) \qquad (7)$$

Shear Stress in y-direction

$$Syc = (Sy+Syb) \qquad (8)$$

Axial Stress in x-direction

$$Sxb = 6*Mx/t^2 \qquad (9)$$

Axial Stress in y-direction

$$Syb = 6*My/t^2 \qquad (10)$$

Axial Stress $Fc=Ft$ is Max (Sx, Sy) (11)

The results in table 2 shows FEA analysis are a good agreement with conceptual design. It also gives an idea that the basic FEA analysis tools that are available in industry can be used for estimation of critical stress/ultimate load capacity for a given cross section with any shape rather than limiting to standard shapes. It also investigated with basic software analysis tool, STAAD.Pro, to find optimal shape of the cross section for given force and area. Cold formed section C10010 with cross sectional area 216 mm^2 taken for this investigation and results are tabulated in table 3. Equivalent shapes RHS and M shapes are analyzed and results are in table 4 and table 5 simultaneously. RHS section shows 37% more compression strength compared to channel section for a given cross sectional area.

TABLE 2: Comparative results of STAAD.Pro Conceptual design Vs. FEA.

Specimen	STAAD.Pro Actual Stress (N/mm^2) Fc	FEA Actual Stress (N/mm^2) Fc
C10010	140.96	121.207
C10012	107.87	100.267
C10015	81.178	78.172
C10019	60.42	60.894

TABLE 3: Maximum Stresses of Channel section C102X51X1 mm.

		Shear		**Membrane**			Bending Moment		
	Plate	*Sqx N/mm2*	*Sqy N/mm2*	*Sx N/mm2*	*Sy N/mm2*	*Sxy N/mm2*	*Mx kNm/m*	*My kNm/m*	*Mxy kNm/m*
Max Qx	2421	**1.156**	0.622	-106.942	-16.692	10.048	0	0	0
Min Qx	2414	-2.069	0.887	-107.22	-15.7	-7.158	-0.001	0	0
Max Qy	2414	-2.069	0.887	-107.22	-15.7	-7.158	-0.001	0	0
Min Qy	2403	-2.069	-0.887	-107.22	-15.7	7.158	-0.001	0	0
Max Sx	2390	-0.859	0.176	-99.583	-0.293	2.26	0	0.001	-0.001
Min Sx	2420	0.001	0.015	**-167.022**	-29.619	-31.979	0	0	0
Max Sy	2241	-0.002	0.001	-114.858	2.048	0.396	0	0	0
Min Sy	32	0.004	0.002	-116.563	-29.8	-1.648	0	0	0
Max Sxy	38	0.713	0.358	-137.487	-22.165	22.311	0	0	0
Min Sxy	2426	-0.001	-0.015	-167.022	-29.619	-31.979	0	0	0
Max Mx	2422	0.111	-0.108	-116.156	-22.287	11.984	**0.001**	**0**	**0**
Min Mx	2413	-0.056	-0.274	-115.355	-21.653	-10.044	-0.002	-0.001	0
Max My	2391	-0.227	-0.164	-100.303	-0.311	0.486	0.001	0.001	0
Min My	2397	0.227	0.164	-100.303	-0.311	0.486	-0.001	-0.001	0
Max Mxy	2379	-0.859	-0.176	-99.583	-0.293	-2.26	0	0.001	0.001
Min Mxy	2390	-0.859	0.176	-99.583	-0.293	2.26	0	0.001	-0.001

TABLE 4. Maximum Stresses of RHS 51X51X1 mm

		Shear		Membrane			Bending Moment		
	Plate	*Sqx N/mm2*	*Sqy N/mm2*	*Sx N/mm2*	*Sy N/mm2*	*Sxy N/mm2*	*Mx kNm/m*	*My kNm/m*	*Mxy kNm/m*
Max Qx	3626	**0.802**	-0.405	-108.966	-14.255	-6.261	0	0	0
Min Qx	2420	-0.802	0.405	-108.966	-14.256	-6.261	0	0	0
Max Qy	2420	-0.802	0.405	-108.966	-14.256	-6.261	0	0	0
Min Qy	2415	-0.802	-0.405	-108.966	-14.255	6.261	0	0	0
Max Sx	2390	-0.196	0.066	-105.921	-0.104	2.237	0	0	0
Min Sx	2432	-0.663	-0.329	**-121.378**	-18.491	13.984	0	0	0
Max Sy	3606	0.003	-0.002	-116.587	1.664	0.886	0	0	0
Min Sy	41	0.023	0.011	-110.188	-23.792	-2.735	0	0	0
Max Sxy	38	0.661	0.327	-121.369	-18.471	13.992	0	0	0
Min Sxy	3027	-0.663	0.329	-121.377	-18.491	-13.983	0	0	0
Max Mx	3625	0.027	0.083	-117.057	-20.48	-6.213	**0.001**	**0**	**0**
Min Mx	2413	-0.027	-0.083	-117.057	-20.48	-6.213	-0.001	0	0
Max My	2390	-0.196	0.066	-105.921	-0.104	2.237	0	0	0
Min My	3620	0.196	-0.066	-105.921	-0.104	2.237	0	0	0
Max Mxy	2391	-0.196	-0.066	-105.921	-0.104	-2.237	0	0	0
Min Mxy	2396	-0.196	0.066	-105.921	-0.104	2.237	0	0	0

TABLE 5 : M Shape section .

		Shear		Membrane			Bending Moment		
	Plate	Sqx N/mm2	Sqy N/mm2	Sx N/mm2	Sy N/mm2	Sxy N/mm2	Mx kNm/m	My kNm/m	Mxy kNm/m
Max Qx	2233	**2.135**	0.622	-99.069	-15.47	8.487	0.002	0.001	0
Min Qx	30	-0.463	-0.201	-132.859	-20.953	14.369	0	0	0
Max Qy	2233	2.135	0.622	-99.069	-15.47	8.487	0.002	0.001	0
Min Qy	2242	1.916	-0.625	-108.62	-15.269	-4.705	0.002	0.001	0
Max Sx	2211	1.186	0.228	-94.178	-0.643	-0.177	0	0	-0.001
Min Sx	2228	0.045	0.007	**-172.587**	-32.279	33.057	0.001	0	0
Max Sy	2174	0.065	-0.019	-112.065	2.824	-1.246	0	0	0
Min Sy	2247	0.034	-0.001	-172.18	-32.498	-33.71	0	0	0
Max Sxy	2228	0.045	0.007	-172.587	-32.279	33.057	0.001	0	0
Min Sxy	2247	0.034	-0.001	-172.18	-32.498	-33.71	0	0	0
Max Mx	2234	-0.13	-0.301	-102.448	-20.018	6.441	**0.003**	**0.001**	**0.001**
Min Mx	61	0.061	0.157	-113.714	-24.035	-6.96	-0.001	0	0
Max My	2234	-0.13	-0.301	-102.448	-20.018	6.441	0.003	0.001	0.001
Min My	2210	0.107	-0.182	-98.654	-0.356	0.556	-0.001	-0.001	0
Max Mxy	2212	-0.173	0.093	-104.2	-6.68	2.694	0	0.001	0.001
Min Mxy	2211	1.186	0.228	-94.178	-0.643	-0.177	0	0	-0.001

Concluding Remarks:

Modern design specifications have taken substantial steps in providing analysis methodology, but these are limited to some standard cross sections. Nowadays, design concepts are becoming increasingly complex due to shape of cross sections. So, when there is complexity due to cross section or structural behavior then finite element analysis and design will be the best practice.

This paper presents finite element investigation on cold formed sections to determine load carrying capacity / critical stresses. To substantiate these results, the standard cold formed steel sections Table 1, designed using STAAD.Pro Structural Analysis and design software per AISI specifications. The FEA results are approximately nearer to AISI conceptual design results. With this, author suggested that “one can follow this approach to design complex shapes of cold formed steel section”.

References:

1. Yu, W.W.1991. Cold-Formed Steel Design, John Wiley & Sons, New York.
2. M.Macdonald, M.A Heiyantuduwa, J.Rohodes. Recent development in the design of cold-formed steel members. Thin-Walled Structures. 46(2008) 1047-1053.
3. Z. Li , B.W. Schafer, Application of the finite strip method in cold-formed steel member design, Journal of Constructional Steel Research 66 (2010) 971_980.
4. Schafer BW. Review: the direct strength method of cold-formed steel member design. Journal of Constructional Steel Research 2008;64:766_78.
5. Cheng Yu_, Yujie Chen, Detailing recommendations for 1.83 m wide cold-formed steel shear walls with steel sheathing, Journal of Constructional Steel Research 67 (2011) 93_101
6. C D. Moen, M.ASCE; and B. W. Schafer, M.ASCE, Direct Strength Method for Design of Cold-Formed Steel Columns with Holes, JOURNAL OF STRUCTURAL ENGINEERING. ASCE / MAY 2011 / 559.
7. Eduardo de Miranda Batista, Effective section method: A general direct method for the design of steel cold-formed members under local–global buckling interaction, Thin-Walled Structures 48 (2010) 345–356.
8. AISI Specification for the Design of Cold-Formed Steel Structural members, 2007 Edition.
9. Schafer, B.W. Distortional buckling of cold-formed steel columns. American Iron and Steel Institute. 2000.
10. Schafer BW, Pekoz T. Direct Strength Prediction of cold-formed steel members using numerical elastic buckling

solutions. In: Shanmugam NE, et al., editors. Thin-walled structures. Amsterdam: Elsevier; 1998.

11. Hancock Gj, Murray TM, Ellifrit DS. Cold-formed steel Structures to the AISI specification. Marcel Dekker, Inc;2001.
12. Nuno Silvestre, Dinar Camotim , Pedro B. Dinis, Post-buckling behaviour and direct strength design of lipped channel columns experiencing local/distortional interaction, volume 73, June 2012, Pages 12–30.